GEOTECHNICAL INVESTIGATION
PROPOSED WEST DUNNE AVENUE WIDENING
BETWEEN PEAK AVENUE AND MONTEREY ROAD
MORGAN HILL, CALIFORNIA

PROJECT 2255E

For

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1. INTRODUCTION

1.1 GENERAL

This report presents the results of our geotechnical investigation for the proposed improvements to a section of West Dunne Avenue between Peak Avenue and Monterey Road in Morgan Hill, California, referred to herein as "study area". The approximate location of the study area is shown on the Vicinity Map included on the Site Plan, Figure 1, of this report. The Site Plan shows the existing improvements and a layout of the preliminary roadway widening.

1.2 PROJECT DESCRIPTION

The portion of West Dunne Avenue to be widened extends from about 125 feet west of Monterey Road to Peak Avenue. The length of roadway to be widened is approximately 2500 feet. We understand the roadway will be widened to accommodate 4 travel lanes plus a middle turn lane. The proposed improvements include new pavements, pavement overlays, retaining walls, curb & gutters, as well as other associated improvements.

The new roadway will include one travel lane in each direction, a middle turn lane, and bike lanes. In addition to pavement widening, City standard sidewalks and curbs and gutters will be constructed, which are currently not present along much of this portion of West Dunne Avenue. As part of the roadway improvements, several residential driveways to private properties will need to be reconfigured to accommodate grade changes at the street. In addition, some retaining walls up to about 8 feet high are anticipated along the roadway.

In general, grade changes along the roadway center line will be less than 6 inches cut or fill. However, because of the widening of the roadway, the cuts and fills are more significant along the margins of the roadway. For most of the roadway, cuts and fills will be less than 18 inches at the roadway margins. Cuts between 2 and 7 feet high will be made at the road margins between Stations 16 and 24 along the north side of the road. Cuts between 1 and 5 feet high will be made at the road margins between Stations 26 and 35 along the north side of the road. Fills between 1 and 3 feet high will be made at the road margins between Stations 18 and 22 along the south side of the road. Retaining walls will likely be required in some of these areas.

If the actual project differs from that described above, Pacific Geotechnical Engineering should be contacted to review our conclusions and recommendations and present any necessary modifications to address the different project development scheme.

1.3 INFORMATION PROVIDED

For this investigation, we were provided with a set of plans prepared by HMH Engineers, dated July 1, 2008, titled "Preliminary Engineering, Santa Teresa/West Dunne." These plans included Sheets PP01, PP02 and PP03, which depict the current roadway and preliminary layout of the widened roadway. These 3 sheets were used as a basis for our analysis and for Figure 1 of this report.

2. INVESTIGATION ACTIVITIES

2.1 OBJECTIVE AND SCOPE OF INVESTIGATION

The objective of this geotechnical investigation was to explore and evaluate subsurface conditions at the site and develop geotechnical recommendations for design and construction of the proposed improvements. Our scope of services for this geotechnical investigation is summarized below.

1. A site reconnaissance to observe site conditions and mark locations of our exploration.
2. Notifying Underground Service Alert of our exploration.
3. Review of in-house geologic and geotechnical information pertaining to the site.
4. Obtain an encroachment permit from the City of Morgan Hill for our subsurface exploration within the existing roadway.
5. Provide traffic control during the field exploration program, consisting of a two-man crew, cones, signs, and lights.
6. Exploration, sampling and classification of subsurface soils by means of 6 exploratory drill holes to between 10 and 20 feet below ground surface in the area of proposed improvements.
7. Coring of the existing pavement in 9 locations to measuring the existing pavement section.
8. Laboratory testing of selected soil samples obtained from the drill holes to evaluate pertinent engineering properties of the samples. Four R-value tests were performed on selected bulk sample.
9. Observations of the existing asphalt pavement surface conditions in the study area.
10. Engineering analysis and evaluation of the field and laboratory testing data.
11. Preparation of this report summarizing the results of our field exploration, laboratory testing, and engineering analyses.

2.2 PAVEMENT SURFACE OBSERVATIONS

We observed the condition of the existing pavement on West Dunne Avenue. Our evaluation of the asphalt concrete pavement (ACP) is in general accordance with Caltrans Highway Design Manual, Section 611.6 - ACP Failure Types. This section of the design manual describes various types of failures in asphalt pavements, such as alligator cracks, transverse and longitudinal cracks, raveling and rutting. A summary of our pavement distress observations is presented in Table 2 in the Appendix.

2.3 SUBSURFACE EXPLORATION

Our subsurface exploration was performed on October 3, 2008, using a truck-mounted Mobile B-53 drill rig equipped with 8-inch diameter hollow-stem augers. Our subsurface exploration program included drilling of six exploratory drill holes (DH-1 through DH-6) excavated to between 10 and 20 feet below ground surface (bgs). The drill holes were located in the field by referencing to existing site features and pacing; therefore, their locations should be considered approximate. The approximate locations of the drill holes are shown on Figure 1. The drill holes were backfilled with cement grout and capped with asphalt cold patch upon completion of drilling and excess soil cuttings were spread on the roadway margins.

Nine (9) pavement probes (PP-1 thru PP-9) were drilled to depths ranging between 1 and 1-½ feet below ground surface. The pavement probes were performed using a Mobile B-53 truck-mounted drilling rig equipped with 8-inch diameter hollow stem augers. The pavement probe locations are shown on Figure 1. The locations of the pavement probes were determined from field measuring and pacing from existing improvements. Their locations should be considered accurate only to the degree implied by the method used. The pavement probes were capped with asphalt cold patch upon completion of drilling.

Six bulk samples of the near-surface soil (Sample B-1 thru B-6) were collected from the proposed pavement area. The approximate locations of the bulk samples are shown on Figure 1.

In the field, our personnel visually classified the materials encountered in the drill holes and maintained a log of each drill hole. Samples were obtained from the drill holes by driving a 2½-inch inside diameter split spoon or a 2-inch outside diameter (1⅜ inch inside diameter) Standard Penetration Test (SPT) sampler up to a depth of 18 inches into the earth material using a 140-pound hammer falling 30 inches. The number of blows required to drive the samplers was recorded for each 6-inch penetration interval. The number of blows required to drive the sampler the last 12 inches, or the interval indicated where higher resistance was encountered, is shown as blows per foot on the drill hole logs. Soil samples were sealed in the field and transported to our laboratory.

Visual classification of soils encountered in our exploratory drill holes was made in general accordance with the Unified Soil Classification System (ASTM D2487 and D2488). The results of our laboratory testing were used to refine our field classifications. Two Keys to Soil Classification, one for fine grained soils and one for coarse grained soils, are included in Appendix A together with the logs of the drill holes.

2.4 LABORATORY TESTING

Laboratory testing was performed on selected soil samples collected from the drill holes. The geotechnical testing included water content, dry density, laboratory compaction, Atterberg Limits and percent passing a No. 200 sieve. Four R-value tests were performed on selected bulk samples. The laboratory test results are presented on the drill hole logs at the corresponding sample depths. The results of the laboratory compaction, Atterberg Limits and R-value tests are shown in Appendix B.

3. FINDINGS

3.1 SURFACE CONDITIONS

The subject section of West Dunne Avenue currently has one traffic lane in the northbound and the southbound directions. The area planned for widening is currently bordered by commercial buildings and residences along most of the south side. Most of the eastern end of the north side of the study area is bounded by residences. However, open fields and widely spaced residences bound the western 3/4 of the north side of West Dunne Avenue in the study area. Natural grades along the road alignment are generally in a south direction. The eastern portion of the existing pavement is relatively level to gently sloping eastward (Station 34 to 40).

The general vicinity in the area of the proposed pavement improvements is gently to moderately sloping. The topography on the site plan provided to us by HMH Engineers indicates the elevations along West Dunne Avenue in the area of proposed widening range from a high of 372.88 feet above sea level (asl) at Station 22+34 to a low of 338.5 feet near to Monterey Road. The roadway grades slope downward toward the east and west from the high point. Gradients along the roadway alignment are mostly sloping gently at inclinations of between ½ to 2 percent. However, on the east and west sides of the topographic high, the gradients are in the range of 3 to 6 percent.

3.2 PAVEMENT SURFACE EVALUATION

We evaluated the condition of the existing pavement on West Dunne Avenue in the study area between Peak Avenue and Monterey Road (STA 15+00 to 40+00) by measuring the thickness of the existing pavement section in 6 drill holes and 9 pavement probes. In addition, we made observations of the condition of the pavement surface in the study area.

3.2.1 Existing Pavement Section

We measured the thickness of the existing pavement section in our 6 drill holes and 9 pavement probes. Our measurements are summarized in Table 1 in the Appendix.

3.2.2 Surface Observations

Our pavement surface evaluation of the asphalt concrete pavement (ACP) in the study area is in general accordance with Caltrans Highway Design Manual, Section 611.6 - ACP Failure Types. This section of the design manual describes various types of failures in asphalt pavements, such as alligator cracks, transverse and longitudinal cracks, raveling and rutting.

We noted significant distress in much of the pavement on West Dunne Avenue. Refer to Table 2 for our observations of the pavement surface along West Dunne Avenue. Our evaluation should not be considered comprehensive, since the primary focus of this investigation is the geotechnical aspects of pavement design. The intent of this evaluation is to make a broad assessment of the pavement in the study area to assist the City in making a cost analysis of pavement repairs. A more comprehensive inventory of should be made cracking and distress and areas requiring patching or sealing.

We characterize the condition of the pavement as follows:

1. In some locations along West Dunne Avenue, the condition of the north side of the street centerline was observed to be different from the south side, in that more distress was present on one side.

2. Most of the distress in the study area consists of light transverse and longitudinal cracks and isolated areas of Type B alligator cracking. Most of the cracking has been sealed, although some of the sealed cracks have reopened and new ones have formed.

> Alligator Cracking
>
> Type A– Initial single or parallel longitudinal fatigue cracks in the wheel paths.
>
> Type B– Interconnected fatigue cracks in the wheel paths.

3. A lot of seams between new and old asphalt, such as at trench backfill patches, seams which have been sealed are typically reopening. In general, cracking along seams between new and old pavement, where it occurs in the wheel lane, is more severe.

4. Generally, the pavement in the study area is in relatively good condition, considering the age of some portions of the pavement. It appears that maintenance (e.g. crack sealing and patching) has helped to extend the life of the pavement.

3.3 SUBSURFACE CONDITIONS

3.3.1 Soil Conditions

In Drill Holes DH-1 and DH-2, we encountered 5 feet or more of alluvium and colluvium consisting of medium dense to dense Clayey Sand and Clayey Gravel. Below 5 feet in DH-2, we encountered greenstone bedrock.

In Drill Holes DH-3, DH-4, DH-5 and DH-6, we encountered colluvium consisting of very stiff to hard Sandy Fat Clay in the upper 5 to 8 feet below the existing pavement surface. The fat clay is underlain by greenstone bedrock that is weathered and intensely fractured to clayey sand and Sandy Fat Clay with a gravel matrix.

3.3.2 Groundwater

Groundwater was not encountered at the time of drilling in any of the drill holes. It should be noted that groundwater depth is subject to seasonal fluctuations depending on rainfall, local irrigation, water recharging program, well pumping, or other factors that may not be evident at the time of our investigation.

4. ANALYSIS, DISCUSSION AND CONCLUSIONS

4.1 GENERAL

Based on the results of our investigation, it is our opinion that, from a geotechnical viewpoint, the proposed street improvements are feasible provided the recommendations presented in this report are incorporated in the project design and construction. Recommendations for project design and construction are presented in the "RECOMMENDATIONS" section of this report.

4.2 SURFACE RUPTURE AND SEISMIC GROUND SHAKING

Because the site is not located within a State of California Earthquake Fault Zone and no mapped active faults are known to cross the site, the probability of ground surface rupture at the site due to displacement along a fault is remote.

The site is in an area of high seismicity. Based on general knowledge of the site seismicity, it should be anticipated that, during its useful life, the proposed development will be subject to at least one severe earthquake (magnitude 7 to 8+) that could cause considerable ground shaking at the site. It is also anticipated that the subject site will periodically experience small to moderate magnitude earthquakes. Proposed improvements should be designed accordingly.

4.3 SOIL EXPANSION POTENTIAL

The surficial layer of soil in much of the proposed pavement and retaining wall areas consists of fat clay of high plasticity. This soil has a high expansion potential.

Expansive soils shrink as the water content decreases such as (during the dry season) and swell as the water content increases (e.g. during the rainy season or by irrigation). The volume change that occurs during this shrinking and swelling process can cause cracking and damage to vehicle pavements, sidewalks, driveways and shallow foundations.

5. RECOMMENDATIONS

5.1 EARTHWORK

Design and construction should comply with the City of Morgan Hill Design Standards and Standard Details for Construction. Construction of street sections is referenced to the Standard Specifications, State of California, Department of Transportation (Caltrans) and the American Society for Testing and Materials (ASTM). The following recommendations are presented as guidelines.

5.1.1 Clearing and Stripping

Site clearing should include removal of debris, deleterious materials, obstructions, structures, foundations, pavements, and stumps and primary roots of trees and brush (roots over 1 inch in diameter or longer than about 3 feet in length). Depressions, voids and holes that extend below proposed finish grade should be cleaned and backfilled with engineered fill compacted to the recommendations in this report.

After clearing, surface vegetation and organic laden soils should be stripped. Organic laden soils are defined as soils with more than 3 percent by weight of organic content. The required stripping depth should be determined in the field by the geotechnical engineer at the time of construction; but for planning purposes, an average stripping depth of 3 inches may be assumed. Stripped material should be removed from the site.

5.1.2 Excavations

Excavations for this project will include general cuts to achieve design grades, trenching for underground utilities, and excavations for retaining wall foundations.

The walls of excavations in the near-surface soil should be able to stand near vertical with minimal bracing, provided proper moisture content in the soil is maintained. Excavations should be constructed in accordance with the current CAL-OSHA safety standards and local jurisdiction. The stability and safety of excavations, braced or unbraced, is the responsibility of the contractor.

Trench excavations adjacent to existing or proposed foundations should be above an imaginary plane having an inclination of 1½:1 (horizontal to vertical) extending down from the bottom edge of the foundations.

5.1.3 Subgrade Preparation

Subgrade soil in areas to receive engineered fill, roadway sections and street improvements should be scarified to a minimum depth of 8 inches, moisture conditioned and compacted to the recommendations given under "Engineered Fill Placement and Compaction." Prepared soil subgrade should be non-yielding when proof-rolled by a fully loaded water truck or equipment of similar weight.

Subgrade preparation should extend a minimum of 3 feet beyond the outermost limits of the proposed construction. After the subgrade has been properly prepared, the areas may be raised to design grades by placement of engineered fill.

Soil with moisture content above optimum value should be anticipated during and shortly after rainy seasons. Where unstable, wet or soft soil is encountered, the soil will require processing before compaction can be achieved. When the construction schedule does not allow for air-drying, other means such as lime or cement treatment of the soil or excavation and replacement with suitable material may be considered. Geotextile fabrics may also be used to help stabilize the subgrade. The method to be used should be determined at the time of construction based on the actual site conditions. We recommend obtaining unit prices for subgrade stabilization during the construction bid process.

5.1.4 Cut and Fill Slopes

Final cut and fill slopes should be constructed at inclinations no steeper than 2:1 (horizontal to vertical). Fill slopes should be overbuilt and cut back to their final configurations.

Proper drainage gradients should be provided to prevent surface runoff from flowing over the crest of slopes which can cause erosion on the slopes. The slopes should be vegetated to help reduce erosion.

5.1.5 Material for Engineered Fill

In general, on-site soils with an organic content of less than 3 percent by weight, free of any hazardous or deleterious materials, and meeting the gradation requirements below may be used as general engineered fill to achieve project grades, except when special material (aggregate base) is required.

In general, engineered fill material should not contain rocks or lumps larger than 3 inches in greatest dimension, should not contain more than 15 percent of the material larger than 1½ inches, and should contain at least 20 percent passing the No. 200 sieve. In addition to these requirements, import fill should have a low expansion potential as indicated by a Plasticity Index of 15 or less, or an Expansion Index of less than 20.

All import fills should be approved by the project geotechnical engineer prior to delivery to the site. At least five (5) working days prior to importing to the site, a representative sample of the proposed import fill should be delivered to our laboratory for evaluation.

5.1.6 Engineered Fill Placement and Compaction

Engineered fill should be placed on properly prepared soil subgrade. Engineered fill should be placed in horizontal lifts each not exceeding 8 inches in thickness and mechanically compacted to the recommendations below at the recommended moisture content. Relative compaction or compaction is defined as the in-place dry density of the compacted soil divided by the laboratory maximum dry density as determined by ASTM Test Method D1557, latest edition, expressed as a percentage. Moisture conditioning of soils should consist of adding water to the soils if they are too dry and allowing the soils to dry if they are too wet. Below is our recommended relative compaction.

Engineered fills consisting of expansive soils, including the on-site fat clay, should be compacted to between 87 and 92 percent relative compaction at moisture content between about 3 and 5 percent above the laboratory optimum value. In pavement areas, the upper 12 inches of subgrade soil should be compacted to a minimum of 95 percent relative compaction with moisture content between 2 and 5 percent above the optimum value.

Engineered fill consisting of soils of low expansion potential, including imported and "non-expansive" fill, should be compacted to a minimum of 90 percent relative compaction at moisture content between about 1 and 3 percent above the laboratory optimum value. In pavement areas, the upper 12 inches of subgrade soil should be compacted to a minimum of 95 percent relative compaction with moisture content between 1 and 3 percent above the optimum value.

Where fill is placed in new City street areas, a minimum of 24 inches below finished grade should be compacted to the recommendations above.

Fills placed on existing slope with an inclination of 5:1 (horizontal to vertical) or steeper should be keyed and benched into the existing slope. Toe keys should extend a minimum of 2 feet into competent material, and have a width of 6 feet or 1½ times the width of the compaction equipment, whichever provides a wider excavation. Toe keys should slope toward their backs with a slope of at least 2 percent. Benches should be created by cutting a minimum of 6 feet into the existing slopes as the new fill is being placed. Vertical spacing of benches should not be more than about 6 feet. Materials excavated from the benches can be mixed with the slope fill and the fill should be compacted to the requirements in this section.

5.1.7 Utility Trench Backfill

Refer to the "Excavations" section of this report regarding utility trench excavations.

Pipe zone backfill, extending from the bottom of the trench to about 1 foot above the top of pipe, should consist of free-draining sand (less than 5% passing a No. 200 sieve) unless concrete is specified. The sand should be compacted to a minimum of 95 percent relative compaction. Above the pipe zone, underground utility trenches may be backfilled with free-draining sand, on-site soil or imported soil. Trench backfill should be compacted to the requirements given in the "Engineered Fill Placement and Compaction" section of this report. Trench backfill should be capped with at least 12 inches of compacted soil (on-site or imported) with a minimum R-value of 15. The backfill material should be placed in lifts not exceeding 6 inches in uncompacted thickness. Thinner lifts may be necessary to achieve the recommended level of compaction of the backfill due to equipment limitations. Compaction should be performed by mechanical means only. Water jetting or flooding to attain compaction of backfill should not be permitted.

5.1.8 Considerations for Soil Moisture and Seepage Control

Subgrade soil and engineered fill should be compacted at moisture content meeting our recommendations. Once compacted, soils should be protected from drying and wetting.

Where concrete sidewalks or pavements abut against landscaped areas, the base rock layer and subgrade soil should be protected against saturation. Water if allowed to seep into the subgrade soil or pavement section could reduce the service life of the improvements. Methods that may be considered to reduce infiltration of water include: 1) subdrains installed behind curbs and slabs in landscape areas; 2) vertical cut-offs, such as a deepened curb section, or equivalent, extending at least 2 inches into the subgrade soil; and 3) use of drip irrigation system for landscape watering.

5.1.9 Wet Weather Construction

If site grading and construction is to be performed during the winter rainy months, the owner and contractors should be fully aware of the potential impact of wet weather. Rainstorms can cause delay to construction and damage to previously completed work by saturating compacted pads or subgrade, or flooding excavations.

Earthwork during rainy months will require extra effort and caution by the contractors. The grading contractor should be responsible to protect his work to avoid damage by rainwater. Standing pools of water should be pumped out immediately. Construction during wet weather conditions should be addressed in the project construction bid documents and/or specifications. We recommend the grading contractor submit a wet weather construction plan outlining procedures they will employ to protect their work and to minimize damage to their work by rainstorms.

5.2 RETAINING WALLS

Retaining walls up to about 8 feet high are proposed along the roadway. We recommend the retaining wall design showing height of wall, backfill material type, drainage details and earth pressures be reviewed by our firm for conformance with our recommendations.

Retaining walls must be designed to resist static earth pressures due to the supported soil and surcharge pressures induced by loads close to the walls. For this project, we recommend the walls be designed using the lateral pressures presented below, which are expressed as equivalent fluid weights.

Soil Pressure	Backfill Slope	Soil/Engineered Fill
Active Soil Pressure	level	55 pcf
	3h:1v	65 pcf
	2h:1v	80 pcf

Active soil pressure may be used for retaining walls where the top of walls is free to deflect and resulting movement of the backfill is acceptable. The soil pressures given above do not include hydrostatic pressures that might be caused by groundwater or water trapped behind the walls.

Backfill against retaining walls should be compacted as discussed in the "Earthwork" Section of this report. Over-compaction should be avoided because increased compaction effort can result in lateral pressures significantly higher than those recommended above. Backfill placed within 3 feet of the walls should be compacted with hand-operated equipment.

A drain should be constructed on the backfill side of the retaining walls to reduce the potential for built-up of hydrostatic pressure. A typical drainage system should consist of a 1 foot wide zone of crushed, free draining gravel (with less than 5 percent fines) wrapped in a geotextile filter fabric (Mirafi 140N or equivalent) or Caltrans Class 2 Permeable Material (Caltrans Standard Specifications, Section 68) immediately adjacent to the walls. Geotextile filter fabric is not required if Class 2 Permeable material is used. A minimum 4-inch diameter, rigid, perforated pipe should be placed in the lower portion of the drainage material to collect discharge water to a storm drain or other discharge facility. The pipe should be PVC Schedule 40 or ABS with an SDR of 35 or better.

Based on the current design, it appears the bottom of the retaining walls may be embedded in greenstone bedrock where they are located on the north side of the roadway and in native soil on the south side. The proposed retaining walls may be supported on conventional footing foundations bearing on undisturbed firm native soil or greenstone bedrock. These footings should be embedded at least 18 inches below rough pad grade or lowest adjacent finish grade, whichever provides a deeper embedment.

Continuous footings bearing on greenstone bedrock may be designed using a net allowable bearing pressure of 2,500 pounds per square foot (psf) for dead plus live loads. This value may be increased by one-third when considering short-term loads such as wind and seismic forces. Reinforcement for the foundations should be determined by the project structural engineer.

Resistance to lateral loads may be developed from a combination of friction between the bottom of foundations and the supporting subgrade, and by passive resistance acting against the vertical sides of the foundations. An ultimate friction coefficient of 0.3 may be used for friction between the foundations and supporting subgrade. Ultimate passive resistance equal to an equivalent fluid weight of 300 pounds per cubic foot (pcf) acting against the embedded sides of the foundations may be used for design purposes. The passive pressure can be assumed to act starting at the top of the lowest adjacent grade in paved areas. In unpaved areas, the passive pressure can be assumed to act starting at a depth of 1 foot below grade. It should be noted that the passive resistance value discussed above is only applicable where the concrete is placed directly against undisturbed soil or engineered fills. Voids created by the use of forms should be backfilled with soil compacted to the requirements given in this report or with concrete.

To maintain foundation support, footings located near utility trenches should be deepened so that the bearing surfaces are below an imaginary plane having an inclination of 1½:1 (horizontal to vertical). This imaginary plane should be drawn extending upward from the bottom edge of the adjacent utility trench.

5.3 SEISMIC PARAMETERS FOR STRUCTURAL DESIGN

Selection of the appropriate seismic design parameters for structures should be made by the project structural engineer after consideration of the site materials, analytical procedures and past performance of similar structures during magnitudes of shaking similar to those expected for this site. The following parameters are developed based on soil data from our subsurface exploration, and Chapter 16 of the CBC (2007). In part, the following values are based on the Java Ground Motion Parameter Calculator, version 5.0.9 (USGS website, Latitude 37.1226 N, Longitude 121.6587 W).

Parameter	Value
Site Class	D (Stiff Soil)
Site Coefficient F_a	1.0
Site Coefficient F_v	1.5
S_{ms} - Modified MCE Spectral Acceleration for Short Period	1.5
S_{m1} - Modified MCE Spectral Acceleration for 1-sec. Period	0.9
S_{DS}	1.0
S_{D1}	0.6

5.4 VEHICLE PAVEMENTS

Vehicle pavements for this project will include new City streets and individual driveways. We understand a design Traffic Index has not yet been established for the subject section of West Dunne Avenue, but will be in the range of between 6 and 9. For the proposed new pavements, we have provided pavement sections for design Traffic Indices of between 6 and 9. The actual pavement section for the proposed driveways should be determined by the project Civil Engineer.

The R-value test results conducted on 4 bulk samples of near-surface soil collected from the site yielded values from 5 to 11. We used an R-value of 5 in our pavement section for the portion of West Dunne Avenue between Peak Avenue and Del Monte Avenue (STA 15+00 to STA 33+00).

Flexible pavement Section Design R-Value = 5 (STA 15+00 to STA 33+00) West Dunne Avenue between Peak Avenue and Del Monte Avenue			
TRAFFIC INDEX	**ASPHALT CONCRETE (inches)**	**CLASS 2 AGGREGATE BASE (inches)**	**TOTAL (inches)**
6.0	4.0*	12.0	16.5
6.5	4.0*	14.0	18.0
7.0	4.0	15.5	19.5
7.5	4.5	17.0	21.0
8.0	4.5	18.5	23.0
8.5	5.0	19.5	24.5
9.0	5.5	20.5	26.0

We used an R-value of 8 in our pavement section for the portion of West Dunne Avenue between Del Monte Avenue and Monterey Road (STA 33+00 to STA 40+00).

Flexible pavement Section Design R-Value = 8 (STA 33+00 to STA 40+00) West Dunne Avenue between Del Monte Avenue and Monterey Road			
TRAFFIC INDEX	**ASPHALT CONCRETE (inches)**	**CLASS 2 AGGREGATE BASE (inches)**	**TOTAL (inches)**
6.0	4.0*	11.0	16.0
6.5	4.0*	13.0	17.5
7.0	4.0	14.5	18.5
7.5	4.5	16.0	20.5
8.0	4.5	17.5	22.0
8.5	5.0	18.5	23.5
9.0	5.5	19.5	25.0

* The City of Morgan Hill has a minimum pavement section of 4 inches of asphalt concrete on 8 inches of Class 2 Aggregate Base for public streets.

Pavement sections should be placed on soil surfaces that have been prepared as outlined in the "Earthwork" section of this report. The full section of aggregate base should be compacted to a minimum of 95 percent relative compaction (ASTM D1557, latest edition). If fill is placed in new City street areas, the upper 24 inches from finished grade should be compacted to the recommendations given under the "Engineered Fill Placement and Compaction" section.

Asphalt Concrete should meet the requirements for 1/2- or 3/4-inch maximum, medium Type A asphalt concrete, Section 39, Caltrans Standard Specifications, latest edition. The Class 2 Aggregate Base material should conform to Section 26 of the Caltrans Standard Specifications.

5.5 SURFACE DRAINAGE

Engineering design of grading and drainage for the project is the responsibility of the project Civil Engineer and should comply with the City's requirements.

6. POST-REPORT GEOTECHNICAL SERVICES

Post-report geotechnical services by Pacific Geotechnical Engineering (PGE), typically consisting of pre-construction design consultations and reviews, construction observation and testing services, are necessary for PGE to confirm the recommendations contained in this report. This report is based on limited sampling and investigation, and by those constraints may not have discovered local anomalies or other varying conditions that may exist on the project site. Therefore, this report is only preliminary until PGE can confirm that actual conditions in the ground conform to those anticipated in the report. Accordingly, as an integral part of this report, PGE recommends post-report geotechnical services to finalize the report and assist the project team during design and construction of the project. PGE requires that it perform these services if it is to remain as the project geotechnical engineer-of-record.

During design, PGE can provide consultation and supplemental recommendations to assist the project team in design and value engineering, especially if the project design has been modified after completion of our report. It is impossible for us to anticipate every design scenario and use of construction materials during preparation of our report. Therefore, retaining PGE to provide post-report consultation will help address design changes, answer questions and evaluate alternatives proposed by the project designers and contractors.

Prior to issuing project plans and specifications for construction bidding purposes, PGE should review the grading, drainage and foundation plans and the project specifications to determine if the intent of our recommendations has been incorporated in these documents. We have found that such a review process will help reduce the likelihood of misinterpretation of our recommendations which may cause construction delay and additional cost.

Construction phase services can include, among other things, the observation and testing during site clearing, stripping, excavation, mass grading, subgrade preparation, fill placement and compaction, backfill compaction, foundation construction and pavement construction activities.

Pacific Geotechnical Engineering would be pleased to provide cost proposals for follow-up geotechnical services. Post-report geotechnical services may include additional field and laboratory services.

7. LIMITATIONS

In preparing the findings and professional opinions presented in this report, we have endeavored to follow generally accepted principles and practices of the engineering geologic and geotechnical engineering professions in the area and at the time our services were performed. No warranty, express or implied, is provided.

The conclusions and recommendations contained in this report are based, in part, on information that has been provided to us. In the event that the general development concept or general location and type of structures are modified, our conclusions and recommendations shall not be considered valid unless we are retained to review such changes and to make any necessary additions or changes to our recommendations. To remain as the project geotechnical engineer-of-record, PGE must be retained to provide geotechnical services as discussed under the Post-report Geotechnical Services section of this report.

Subsurface exploration is necessarily confined to selected locations and conditions may, and often do, vary between these locations. Should conditions different from those described in this report be encountered during project development, PGE should be consulted to review the conditions and determine whether our recommendations are still valid. Additional exploration, testing, and analysis may be required for such evaluation.

Should persons concerned with this project observe geotechnical features or conditions at the site or surrounding areas which are different from those described in this report, those observations should be reported immediately to Pacific Geotechnical Engineering for evaluation.

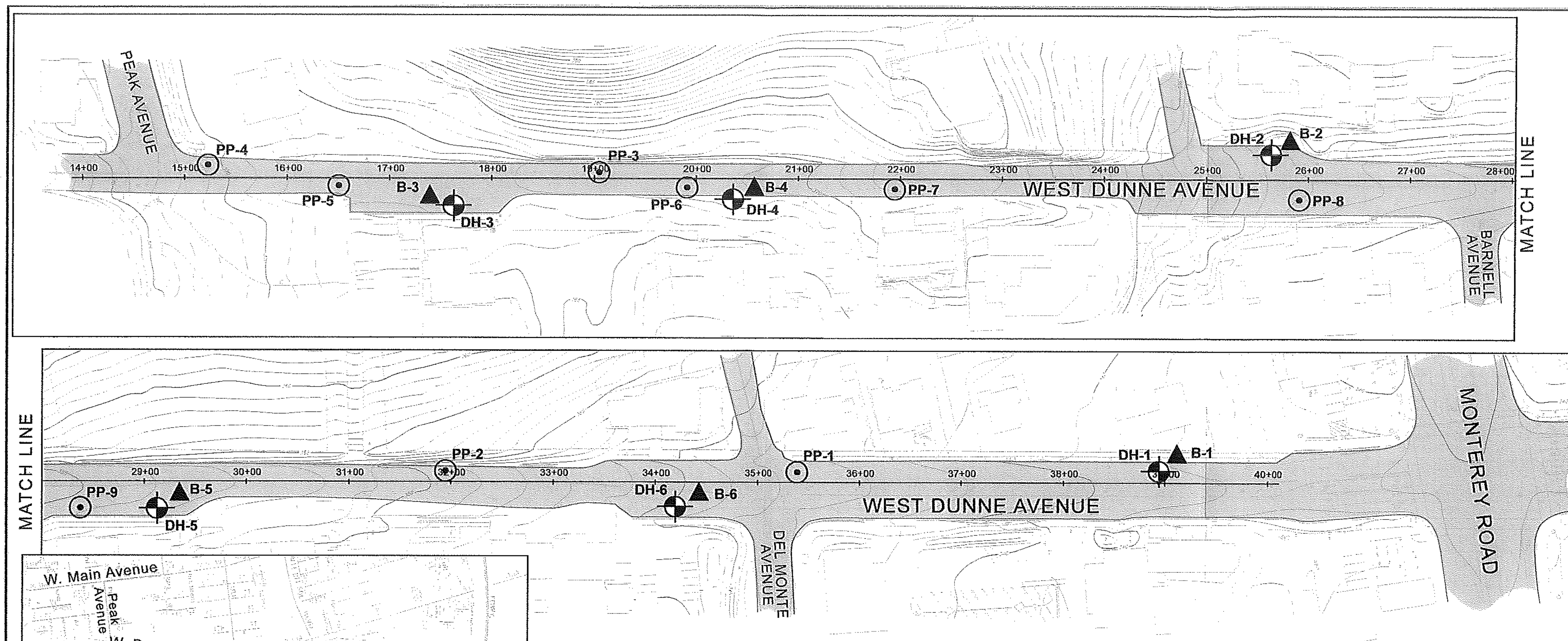
It is important that the information in this report be made known to the design professionals involved with the project, that our recommendations be incorporated into project drawings and documents, and that the recommendations be carried out during construction by the contractor and subcontractors. It is not the responsibility of Pacific Geotechnical Engineering to notify the design professionals and the project contractors and subcontractors.

The findings, conclusions and recommendations presented in this report are applicable only to the specific project development on this specific site. These data should not be used for other projects, sites or purposes unless they are reviewed by PGE or a qualified geotechnical professional.

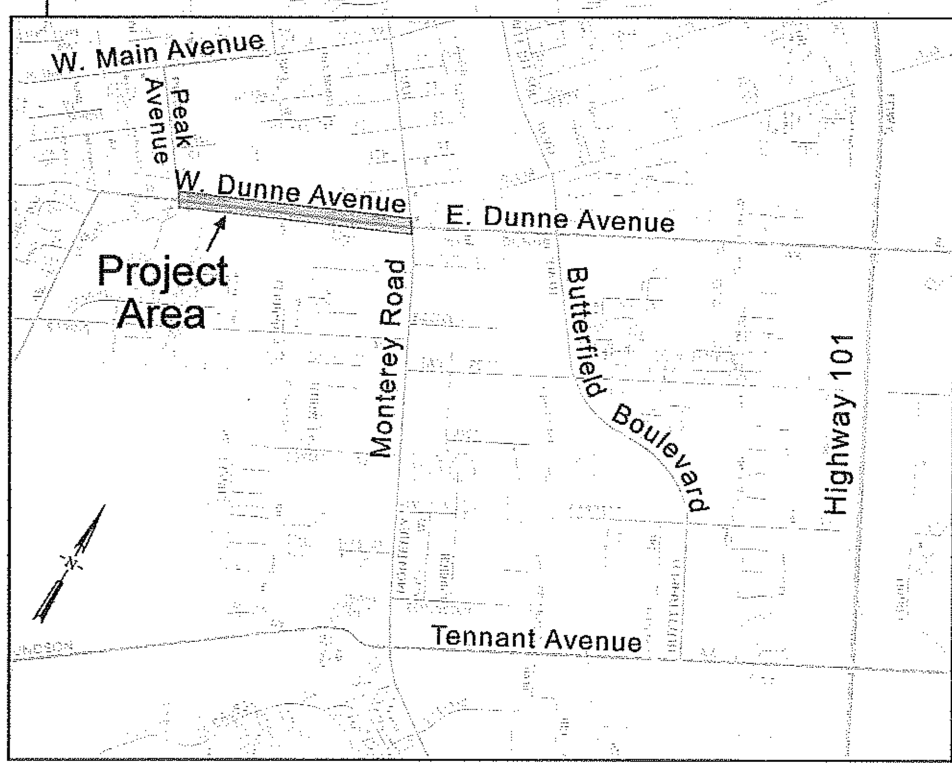
Report prepared by,

PACIFIC GEOTECHNICAL ENGINEERING

Daniel J. Peluso
Geotechnical Engineer 2367



BASE: AutoCAD file titled "51100EG.dwg" prepared by HMH Engineers, received electronically.



VICINITY MAP - no scale

EXPLANATION

DH-6 Exploratory drill hole

B-6 Bulk sample

PP-9 Pavement probe

PACIFIC GEOTECHNICAL ENGINEERING	DATE	SITE PLAN	FIGURE
	NOVEMBER 2008	WEST DUNNE AVENUE WIDENING WEST DUNNE AVENUE from PEAK AVENUE to MONTEREY ROAD MORGAN HILL, CALIFORNIA	1
			PROJECT
			2255E

TABLE 1

West Dunne Avenue Widening
Pavement Evaluation for New Improvements
Logs of Pavement Probes and Drill Hole Pavement Measurements
(In order of increasing Station Number)

Pavement Probe No.	PP-4	PP-5	**DH-3**	PP-3	PP-6	**DH-4**	PP-7	**DH-2**	PP-8	PP-9	**DH-5**	PP-2	**DH-6**	PP-1	**DH-1**
Location	STA 15+25 10L	STA 16+50 10R	STA 17+60 20R	STA 19+05 8L	STA 19+95 10R	STA 20+40 20R	STA 21+95 10R	STA 25+60 15L	STA 25+95 15R	STA 28+40 15R	STA 29+10 15r	STA 31+95 8L	STA 34+20 15R	STA 35+35 10L	STA 38+95 10L
Asphalt Thickness (inches)	2" AC 6" PCC	7	6	8	8	1	6	6	6-½	7	8-½	5	5	6	6
Aggregate Base Thickness (inches)	12	4	12	4	3	0	10	12	4	5	0	6	5	5	6
Subgrade Soil Description	-	-	Fat Clay with Sand (CH), Black moist, very stiff	-	-	Sandy Fat Clay (CH), Dark brown, moist, hard	-	Clayey Sand (SC), Dark brown, moist, medium dense	-	-	Sandy Fat Clay (CH), Reddish brown, moist, hard	-	Sandy Fat Clay (CH), Dark reddish brown, moist, hard	-	Clayey Sand (SM), Brown, moist, dense
Approximate Equivalent Traffic Index Based on R-value = 5	8.0	5.5	7.0	6.0	6.0	< 4.0 Not in Travel Way	6.5	7.0	5.5	6.0	5.5	5.5	5.0	5.5	5.5

Notes:

1. Station locations are approximate
2. Equivalent traffic index should not be considered a validation that the existing pavement meets that traffic index. It is intended as a guide to evaluate exisitng pavement section thickness and the possible need for overlays.
3. Equivalent traffic index indicated above does not consider degradation of the pavement due to age and distress. A factor or coefficient should be applied to the pavement section thickness to account for fatigue.

TABLE 2

West Dunne Avenue Widening
Pavement Improvements
West Dunne Avenue Pavement Evaluation

STATION	Observations	Preliminary Recommendation
16+50 to 20+00, 0 - 10' L	Light to medium longitudinal and transverse cracking	Seal cracks
16+00 to 16+50, 0 - 12' R	Fine raveling in isolated areas	Overlay
19+00 to 20+00, 0 - 10' R	Light to medium longitudinal and transverse cracking, minor Type B alligator cracking	Seal cracks, patch alligatoring
20+00 to 22+00, 0 - 12' R	Light to medium longitudinal and transverse cracking	Seal cracks
22+50, 10' R and 3' L	Localized minor Type B alligator cracks	Patch cracks
22+00 to 25+00, both sides	Light to medium longitudinal and transverse cracking	Seal cracks
24+10	Localized minor Type B alligator cracks	Patch cracks
25+00 to 27+10, 0 - 8' L	Light longitudinal and transverse cracking, isolated Type A and B alligator cracks, isolated fine raveling	Seal cracks, patch alligatoring, overlay raveling
28+00 to 28+90, 0 - 12' L	Light to medium longitudinal and transverse cracking, Localized minor Type B alligator cracks	Seal cracks, patch alligatoring
29+20 to 30+00, 0 - 10' L	Light to medium longitudinal and transverse cracking, primarily near roadway center line	Seal cracks
30+00 to 31+00, 0 - 10' L	Minor longitudinal and transverse cracking, Localized minor Type B alligator cracks	Seal cracks, patch alligatoring
31+30 to 32+60, 0 - 12' L	Moderate longitudinal and transverse cracking, Localized minor Type B alligator cracks at STA 31+80 and 32+00	Seal cracks, patch alligatoring
32+00, 0 - 10' R	Pavement cracking associated with existing patches	Seal cracks
33+00 to 35+00, both travel ways	Moderate longitudinal and minor transverse cracking, sealed in travel way	Seal open cracks
33+90 to 34+00, 8' R	Type A alligator cracks associated with longitudinal cracks near center line	Seal cracks
35+50 to 37+00, center	Longitudinal cracks near center line, minor transverse cracks	Seal cracks
38+00 to 38+50, center	Type A alligator cracks associated with longitudinal cracks near center line	Seal cracks
38+20 to 38+50, 10 - 15' R	Hummocky pavement surface, patches near by, may be due to water line break saturating subgrade	Replace pavement
38+50 to 40+00, 0 - 15' R	Localized minor Type B alligator cracks; appears to be associated with paving over trench backfill or previous patch	Patch alligatoring

APPENDIX A
KEYS TO SOIL CLASSIFICATION
AND
DRILL HOLE LOGS

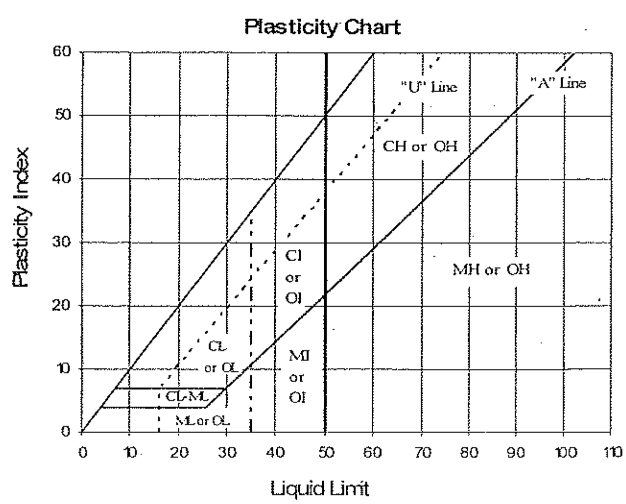
KEY TO SOIL CLASSIFICATION - FINE GRAINED SOILS
(50% OR MORE IS SMALLER THAN NO. 200 SIEVE SIZE)
(modified from ASTM D2487 to include fine grained soils with intermediate plasticity)

MAJOR DIVISIONS			GROUP SYMBOLS	GROUP NAMES
SILTS AND CLAYS (Liquid Limit less than 35) Low Plasticity	Inorganic	PI < 4 or plots below "A" line	ML	Silt, Silt with Sand or Gravel, Sandy or Gravelly Silt, Sandy or Gravelly Silt with Sand or Gravel
	Inorganic	PI > 7 or plots on or above "A" line	CL	Lean Clay, Lean Clay with Sand or Gravel, Sandy or Gravelly Lean Clay, Sandy or Gravelly Lean Clay with Sand or Gravel
	Inorganic	PI between 4 and 7	CL-ML	Silty Clay, Silty Clay with Sand or Gravel, Sandy or Gravelly Silty Clay, Sandy or Gravelly Silty Clay with Sand or Gravel
	Organic	See footnote 3	OL	Organic Silt (below "A" Line) or Organic Clay (on or above "A" Line) (1,2)
SILTS AND CLAYS (35 ≤ Liquid Limit < 50) Intermediate Plasticity	Inorganic	PI < 4 or plots below "A" line	MI	Silt, Silt with Sand or Gravel, Sandy or Gravelly Silt, Sandy or Gravelly Silt with Sand or Gravel
	Inorganic	PI > 7 or plots on or above "A" line	CI	Clay, Clay with Sand or Gravel, Sandy or Gravelly Clay, Sandy or Gravelly Clay with Sand or Gravel
	Organic	See footnote 3	OI	Organic Silt (below "A" Line) or Organic Clay (on or above "A" Line) (1,2)
SILTS AND CLAYS (Liquid Limit 50 or greater) High Plasticity	Inorganic	PI plots below "A" line	MH	Elastic Silt, Elastic Silt with Sand or Gravel, Sandy or Gravelly Elastic Silt, Sandy or Gravelly Elastic Silt with Sand or Gravel
	Inorganic	PI plots on or above "A" line	CH	Fat Clay, Fat Clay with Sand or Gravel, Sandy or Gravelly Fat Clay, Sandy or Gravelly Fat Clay with Sand or Gravel
	Organic	See note 3 below	OH	Organic Silt (below "A" Line) or Organic Clay (on or above "A" Line) (1,2)

1. If soil contains 15% to 29% plus No. 200 material, include "with sand" or "with gravel" to group name, whichever is predominant.
2. If soil contains ≥30% plus No. 200 material, include "sandy" or "gravelly" to group name, whichever is predominant. If soil contains ≥15% of sand or gravel sized material, add "with sand" or "with gravel" to group name.
3. Ratio of liquid limit of oven dried sample to liquid limit of not dried sample is less than 0.75.

CONSISTENCY	UNCONFINED SHEAR STRENGTH (KSF)	STANDARD PENETRATION (BLOWS/FOOT)
VERY SOFT	< 0.25	< 2
SOFT	0.25 – 0.5	2 – 4
FIRM	0.5 – 1.0	5 – 8
STIFF	1.0 – 2.0	9 – 15
VERY STIFF	2.0 – 4.0	16 – 30
HARD	> 4.0	> 30

MOISTURE	CRITERIA
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp, but no visible water
Wet	Visible free water, usually soil is below the water table



PACIFIC GEOTECHNICAL ENGINEERING

KEY TO SOIL CLASSIFICATION – COARSE GRAINED SOILS

(MORE THAN 50% IS LARGER THAN NO. 200 SIEVE SIZE)

(modified from ASTM D2487 to include fines with intermediate plasticity)

MAJOR DIVISIONS			GROUP SYMBOLS	GROUP NAMES[1]
GRAVELS (more than 50% of coarse fraction is larger than No. 4 sieve size)	Gravels with less than 5% fines	Cu ≥ 4 and 1 ≤ Cc ≤ 3	GW	Well Graded Gravel, Well Graded Gravel with Sand
		Cu < 4 and/or 1 > Cc > 3	GP	Poorly Graded Gravel, Poorly Graded Gravel with Sand
	Gravels with 5% to 12% fines	ML, MI or MH fines	GW-GM	Well Graded Gravel with Silt, Well Graded Gravel with Silt and Sand
			GP-GM	Poorly Graded Gravel with Silt, Poorly Graded Gravel with Silt and Sand
		CL, CI or CH fines	GW-GC	Well Graded Gravel with Clay, Well Graded Gravel with Clay and Sand
			GP-GC	Poorly Graded Gravel with Clay, Poorly Graded Gravel with Clay and Sand
	Gravels with more than 12% fines	ML, MI or MH fines	GM	Silty Gravel, Silty Gravel with Sand
		CL, CI or CH fines	GC	Clayey Gravel, Clayey Gravel with Sand
		CL-ML fines	GC-GM	Silty Clayey Gravel; Silty, Clayey Gravel with Sand
SANDS (50% or more of coarse fraction is smaller than No. 4 sieve size)	Sands with less than 5% fines	Cu ≥ 6 and 1 ≤ Cc ≤ 3	SW	Well Graded Sand, Well Graded Sand with Gravel
		Cu < 6 and/or 1 > Cc > 3	SP	Poorly Graded Sand, Poorly Graded Sand with Gravel
	Sands with 5% to 12% fines	ML, MI or MH fines	SW-SM	Well Graded Sand with Silt, Well Graded Sand with Silt and Gravel
			SP-SM	Poorly Graded Sand with Silt, Poorly Graded Sand with Silt and Gravel
		CL, CI or CH fines	SW-SC	Well Graded Sand with Clay, Well Graded Sand with Clay and Gravel
			SP-SC	Poorly Graded Sand with Clay, Poorly Graded Sand with Clay and Gravel
	Sands with more than 12% fines	ML, MI or MH fines	SM	Silty Sand, Silty Sand with Gravel
		CL, CI or CH fines	SC	Clayey Sand, Clayey Sand with Gravel
		CL-ML fines	SC-SM	Silty, Clayey Sand; Silty, Clayey Sand with Gravel

US STANDARD SIEVES 3 Inch ¾ Inch No. 4 No. 10 No. 40 No. 200

	COARSE	FINE	COARSE	MEDIUM	FINE	
COBBLES & BOULDERS	GRAVELS		SANDS			SILTS AND CLAYS

RELATIVE DENSITY (SANDS AND GRAVELS)	STANDARD PENETRATION (BLOWS/FOOT)
Very Loose	0 - 4
Loose	5 – 10
Medium Dense	11 – 30
Dense	31 - 50
Very Dense	50+

1. Add "with sand" to group name if material contains 15% or greater of sand-sized particle. Add "with gravel" to group name if material contains 15% or greater of gravel-sized particle.

MOISTURE	CRITERIA
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp, but no visible water
Wet	Visible free water, usually soi is below the water table

PACIFIC GEOTECHNICAL ENGINEERING

ROCK QUALITY DESCRIPTIONS

HARDNESS**

Very Hard	Cannot be scratched with knife or sharp pick. Breaking of hand specimens requires several hard blows of the geologist's pick
Hard	Can be scratched with knife or pick only with difficulty. Hard blow with hammer required to break sample.
Moderately Hard	Can be scratched with knife or pick. Gouges or grooves to ½ inch can be excavated by hard blow of point of a geologist's pick. Hand specimens broken with moderate blow.
Medium	Can be grooved or gouged 1/16 inch deep by firm pressure on knife or pick point. Can be excavated in small chips about 1 inch maximum in dimension by hard blows of the point of a geologist's pick.
Soft	Can be grooved or gouged readily with knife or pick point. Can be excavated in chips to pieces several inches in size by moderate blows of a pick point. Small pieces can be broken by finger pressure,
Very Soft	Can be carved with knife. Can be excavated readily with point of pick. Pieces one inch or more thickness can be broken with finger pressure. Can be scratched readily by finger nail.

WEATHERING**

Fresh or Unweathered	Rock fresh, crystals bright, few joints and fractures may show slight staining. Rock rings under hammer if crystalline.
Very Slight	Rock generally fresh, fractures and joints stained, some joints may show thin clay coatings, crystals in broken face show bright. Rock rings under hammer if crystalline.
Slight	Rock generally fresh, joints and fractures stained, and discoloration extends into rock up to 1 inch. Joints may contain clay. In granitic rock, some occasional feldspar crystals are dull and discolored. Crystalline rocks ring under hammer.
Moderate	Significant portions of rock show discoloration and weathering effects. In granitic rock, most feldspars are dull and discolored; some show clay. Rock has dull sound under hammer and shows significant loss of strength as compared with fresh rock.
Moderately Severe	All rock except quartz discolored or stained. In granitic rock, all feldspars dull and discolored and majority show kaolinization. Rock shows severe loss of strength and can be excavated with geologist's pick. Rock goes "clunk" when struck.
Severe	All rock except quartz discolored or stained. Rock "fabric" clear and evident, but reduced in strength to strong soil. In granitic rock, all feldspars kaolinized to some extent. Some fragments of strong rock usually left.
Very Severe	All rock except quartz discolored or stained. Rock "fabric" discernible, but mass effectively reduced to "soil" with only fragments of strong rock remaining.
Complete	Rock reduced to "soil." Rock "fabric" not discernible or discernible only in small scattered locations. Quartz may be present as dikes or stringers.

FRACTURE DIMENSIONS*

Fracture	Block Size (or Spacing[1])
Crushed	~5 microns to 0.1 ft
Intensely	0.05 to 0.1 ft
Closely	0.1 to 0.5 ft
Moderately	0.5 to 1.0 ft
Slightly	1.0 to 3.0 ft
Massive	3.0 ft and larger

1 Average distance between adjacent fractures
* Source of data unknown
** Source of data: "Subsurface Investigaiton for Design and Constructio of Foundation Buildings," (1976) American Society of Civil Engineers, Manuals and Reports on Engineering Practice – No. 5

DATE: 10/3/2008	LOG OF EXPLORATORY DRILL HOLE	DH- 1

PROJECT NAME: West Dunne Avenue Widening **PROJECT NUMBER:** 2255 E

DRILL RIG: Mobile B53 with 140# down hole hammer & wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: Initial: --- Final: ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
6 inches of asphalt over 6 inches base rock												
ALLUVIUM: CLAYEY SAND: Brown (7.5YR 4/4), dry to moist, dense; fine to medium sand; trace of coarse sand to fine gravel; grades coarser with depth	SC	1	S									
		2	D D	58	4.5+			10		112		
CLAYEY SAND to CLAYEY SAND with GRAVEL: Brown (10YR 4/3), moist, dense; fine to medium subangular to subrounded sand and gravel	SC	3	S									
		4	D D	72				10		120		
		5	S									
		6	D D	88/11"								
		7										
		8										
CLAYEY GRAVEL with SAND: Brown (10YR 4/3), moist, dense; fine to medium subangular to subrounded sand and gravel	GC	9	S D D	58								
		10										
BOTTOM OF HOLE = 10 Feet No Groundwater Encountered		11										
		12										
		13										
		14										
		15										
		16										
		17										
		18										
		19										
		20										

DATE: 10/3/2008	LOG OF EXPLORATORY DRILL HOLE	DH- 2

PROJECT NAME: West Dunne Avenue Widening **PROJECT NUMBER:** 2255 E

DRILL RIG: Mobile B53 with 140# down hole hammer & wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: Initial: --- Final: ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
6 inches of asphalt over 12 inches baserock		1	S									
COLLUVIUM: CLAYEY SAND to CLAYEY SAND with GRAVEL: Dark brown (10YR 3/3), moist, medium dense; fine to coarse sand; fine to coarse angular to rounded gravel and minor cobbles grades into bedrock	SC	2	D D	41								
		3	S									
		4	D D	23				20		112		
BEDROCK: GREENSTONE: Completely weathered to **CLAYEY SAND/SANDY CLAY:** Brown (10YR 4/3), moist, dense; fine to medium subangular to subrounded sand and gravel	SC to CI	5	S									
		6	D D	68	4.5+							
		7										
		8										
		9	S									
		10	D D	51								
BOTTOM OF HOLE = 10 Feet No Groundwater Encountered		11										
		12										
		13										
		14										
		15										
		16										
		17										
		18										
		19										
		20										

DATE: 10/3/2008	LOG OF EXPLORATORY DRILL HOLE	DH- 3

PROJECT NAME: West Dunne Avenue Widening **PROJECT NUMBER:** 2255 E

DRILL RIG: Mobile B53 with 140# down hole hammer & wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: Initial: --- Final: ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
6 inches of asphalt over 12 inches baserock		1										
COLLUVIUM: FAT CLAY with SAND: Black (7.5YR 2.5/1), moist, very stiff; mostly fine sand	CH	2	S D D	29	3.75			20		105		
		3										
		4	S D D	22	4.25			22		99		
		5										
		6	S D D	34								
gradational contact ± .5 feet		7										
BEDROCK: GREENSTONE: Completely weathered to **CLAYEY SAND/SANDY CLAY:** Brown (7.5YR 4/4), moist, very stiff/medium dense		8										
		9	S D D	43								
		10										
BOTTOM OF HOLE = 10 Feet No Groundwater Encountered		11										
		12										
		13										
		14										
		15										
		16										
		17										
		18										
		19										
		20										

DATE: 10/3/2008	LOG OF EXPLORATORY DRILL HOLE	DH- 4

PROJECT NAME: West Dunne Avenue Widening **PROJECT NUMBER:** 2255 E

DRILL RIG: Mobile B53 with 140# down hole hammer & wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: Initial: --- Final: ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
1 inch asphalt												
COLLUVIUM: SANDY FAT CLAY: Dark brown (7.5YR 2.5/2), moist, hard; mostly fine to medium angular sand	CH	1	S									
		2	D D	24	4.5+	61	55	20	30	103		
		3	S									
		4	D D	15	4.25			22		103		
BEDROCK: GREENSTONE: Completely weathered to **SANDY FAT CLAY:** Strong brown (7.5YR 4/6), moist, very stiff; mostly fine angular sand; minor medium to coarse angular sand	CH	5	S									
		6	D D	21	2.75 4.0							
		7										
GREENSTONE: Strong brown, soft due to fracture and weathering; no fracture observed in samples; very severely weathered		8										
		9	S D D	74								
		10										
		11										
		12										
		13										
Brown to strong brown; crushed; abundant manganese stains; soft; fragments are hard; moderately to severely weathered		14	S I I	35								
		15										
		16										
		17										
		18										
moderately weathered		19	S I	50/6"								
BOTTOM OF HOLE = 19.5 Feet No Groundwater Encountered		20										

DATE: 10/3/2008	LOG OF EXPLORATORY DRILL HOLE	DH- 5

PROJECT NAME: West Dunne Avenue Widening **PROJECT NUMBER:** 2255 E

DRILL RIG: Mobile B53 with 140# down hole hammer & wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: Initial: --- Final: ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
8-½ inches asphalt; no baserock												
		1	S									
COLLUVIUM: SANDY FAT CLAY to CLAYEY SAND: Reddish brown (5YR 4/4); moist, hard; mostly fine to medium angular to rounded sand; minor coarse sand to fine gravel	CH	2	D	9	4.5							
			D		4.5			17		111		
		3	S									
		4	D	19	4.5+							
			D		4.5+			19		105		
		5	S									
		6	D	41	4.5+							
			D		4.5+							
		7										
		8										
BEDROCK: GREENSTONE: Variably mottled pale brown and strong brown, moist, soft due to weathering; abundant manganese staining; very severely weathered; abundant clay; crushed; relict brittle structure observed in samples		9	S									
			D	45								
		10	D									
		11										
		12										
		13										
		14	S									
			D	51								
		15	D									
		16										
		17										
		18										
		19	S									
BOTTOM OF HOLE = 19.5 Feet No Groundwater Encountered			I	50/6"								
		20										

DATE: 10/3/2008	LOG OF EXPLORATORY DRILL HOLE	DH- 6

PROJECT NAME: West Dunne Avenue Widening **PROJECT NUMBER:** 2255 E

DRILL RIG: Mobile B53 with 140# down hole hammer & wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: Initial: --- Final: ---

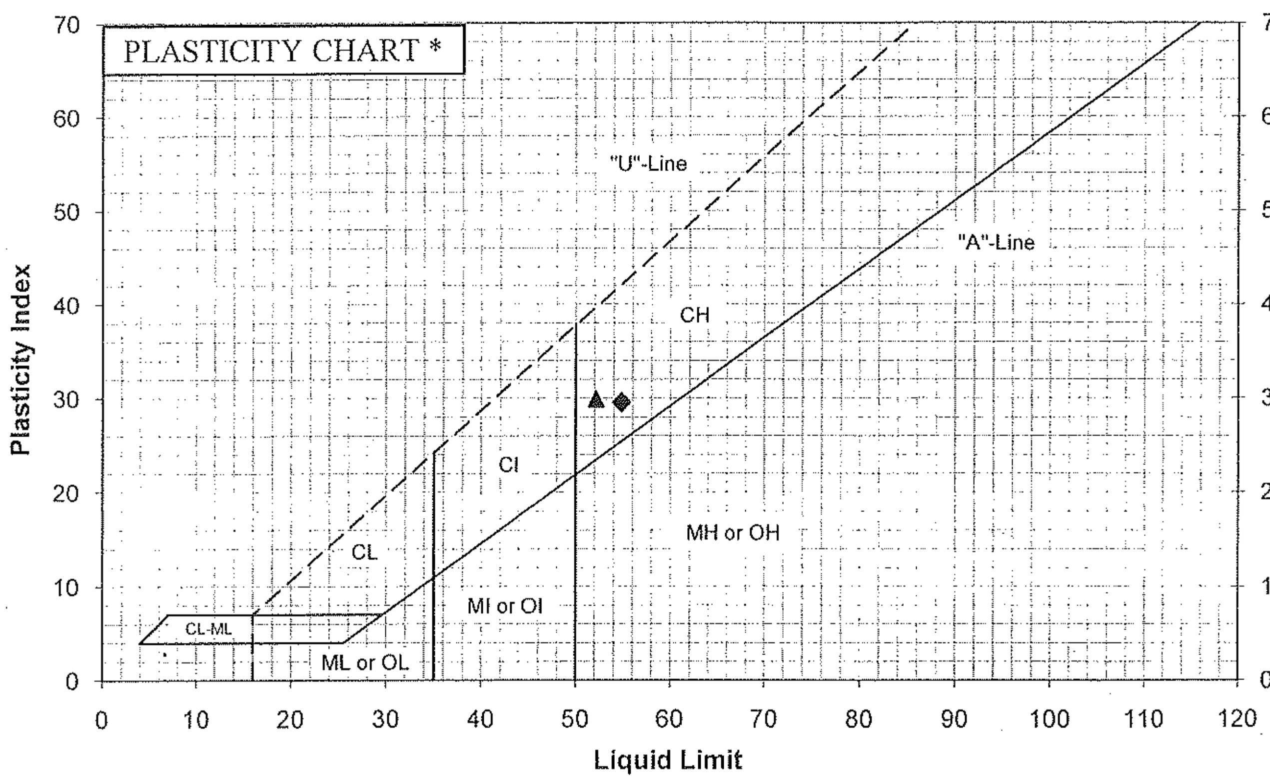
DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
5 inches asphalt over 5 inches baserock												
COLLUVIUM: SANDY FAT CLAY: Dark reddish brown (5YR 5/4); moist, dense; mostly fine to medium angular to rounded sand; minor coarse sand to fine gravel	CH	1	S									
		2	D	20	4.5							
			D		4.5	65	52	22	30	109		
		3	S									
		4	D	39	4.5+							
			D		4.5+			19		110		
		5	S									
		6	D	66	4.5+							
			D		4.5+							
		7										
BEDROCK: GREENSTONE: Completely weathered to **SANDY FAT CLAY/CLAYEY SAND:** Yellowish red (5YR 4/6), moist, hard/dense; mostly fine sand	CH-SC	8										
		9	S									
			D	60	4.5+							
		10	D		4.5+							
		11										
		12										
		13										
		14	S									
			I	33								
		15	I									
		16										
		17										
		18										
		19	S									
BOTTOM OF HOLE = 20 Feet No Groundwater Encountered			I	52								
		20	I									

PACIFIC GEOTECHNICAL ENGINEERING

APPENDIX B
LABORATORY TEST RESULT SHEETS

ATTERBERG LIMITS TEST RESULTS

PROJECT NAME	Dunne Ave. Widening		PROJECT No. 2255 E	
DATE OF TEST	10/30/2008	10/31/2008		
KEY SYMBOL	◆	▲		
DRILL HOLE No.	4	6		
DEPTH (ft)	2	2		
NATURAL WATER CONTENT (%)	20	22		
% Retained No. 40 SIEVE (Est.)	20	15		
% PASSING No. 200 SIEVE	61	65		
LIQUID LIMIT	55	52		
PLASTIC LIMIT	25	22		
PLASTICITY INDEX	30	30		
CLASSIFICATION SYMBOL	CH	CH		



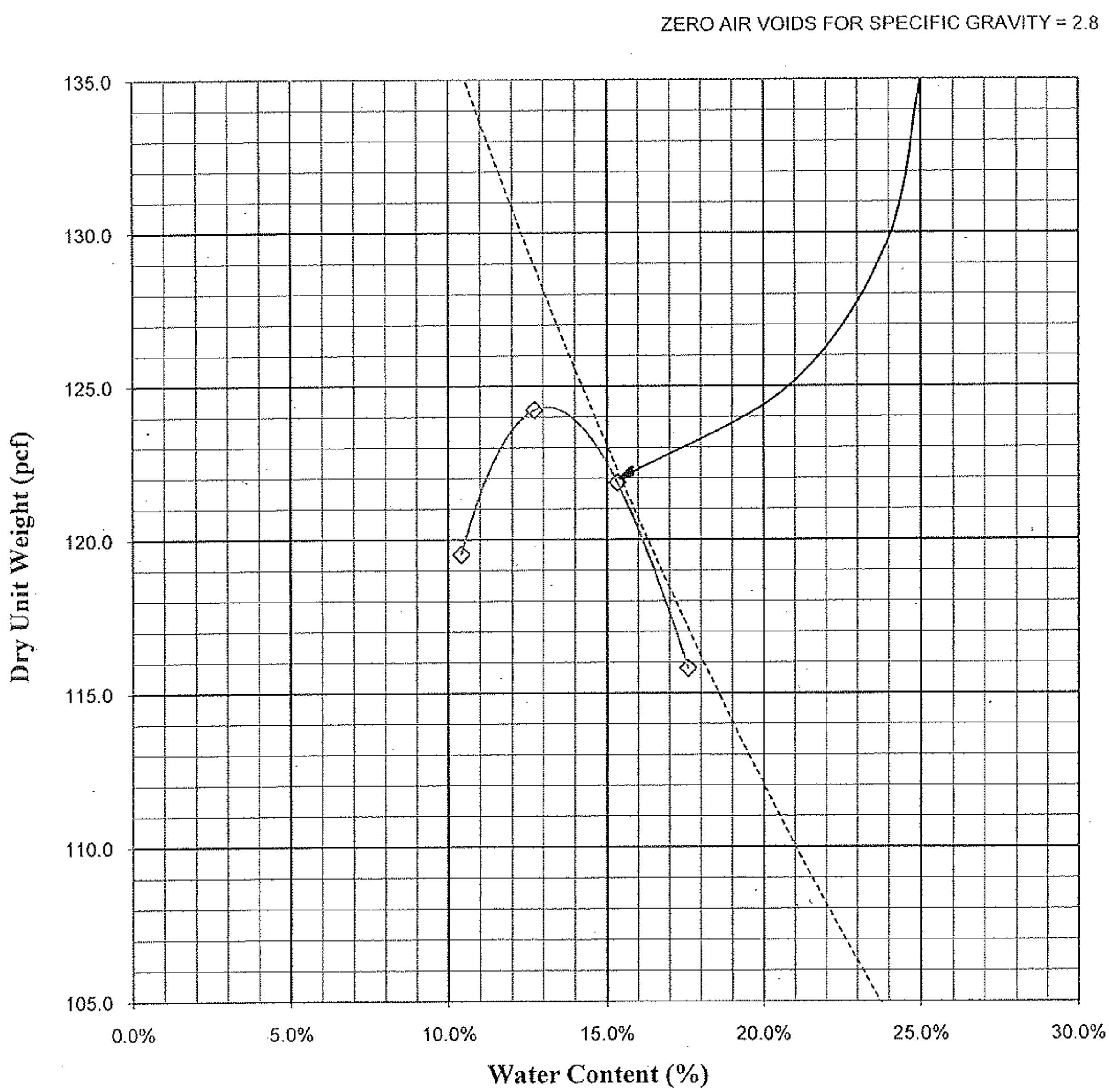
* Based on the Unified Soil Classification System modified to incorporate the "intermediate" classifications CI, MI, and OI for soils with liquid limits between 35 and 50. In the unmodified Unified Soil Classification System, such soils would be classified as CL, ML and OL, respectively.

PACIFIC GEOTECHNICAL ENGINEERING

Rev030106

COMPACTION TEST RESULTS

PROJECT NAME: Dunne Ave			**PROJ. No.:** 2255 E
TEST & METHOD ASTM D1557-91 B	**SAMPLE:** DH 2 B2	**DEPTH:** 2-5'	**DATE:** 10/13/2008
DESCRIPTION OF SOIL: CLAYEY SAND: Dark brown (10YR 3/3), dry, 30-40% clay fines; minor angular fine gravel			
MAXIMUM DRY UNIT WEIGHT(pcf): 124.5		**OPTIMUM WATER CONTENT(%):** 13.0	

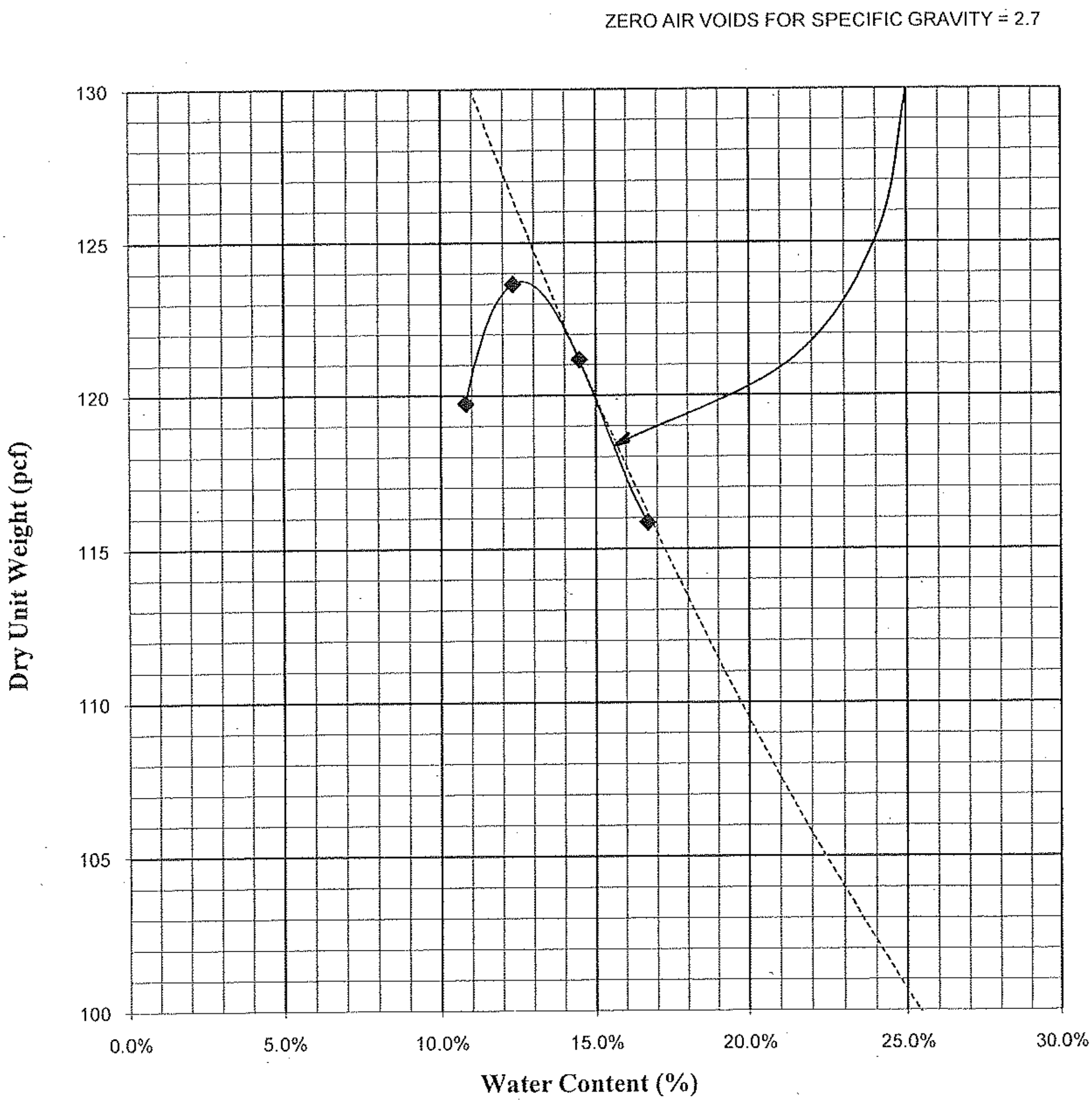


PACIFIC GEOTECHNICAL ENGINEERING

Rev010306

COMPACTION TEST RESULTS

PROJECT NAME: Dunne Ave			PROJ. No.: 2255/E
TEST & METHOD ASTM D1557-91 B	SAMPLE: B-5	DEPTH: 2.0-5.0'	DATE: 10/13/2008
DESCRIPTION OF SOIL: SANDY FAT CLAY: Dark Brown (10YR3/3), dry; 30-40% fine to corse sand. Sample has a volatile organic odor			
MAXIMUM DRY UNIT WEIGHT(pcf): 123.5		OPTIMUM WATER CONTENT(%): 12.5	



PACIFIC GEOTECHNICAL ENGINEERING

COOPER TESTING LABORATORY

R-value Test Report (Caltrans 301)

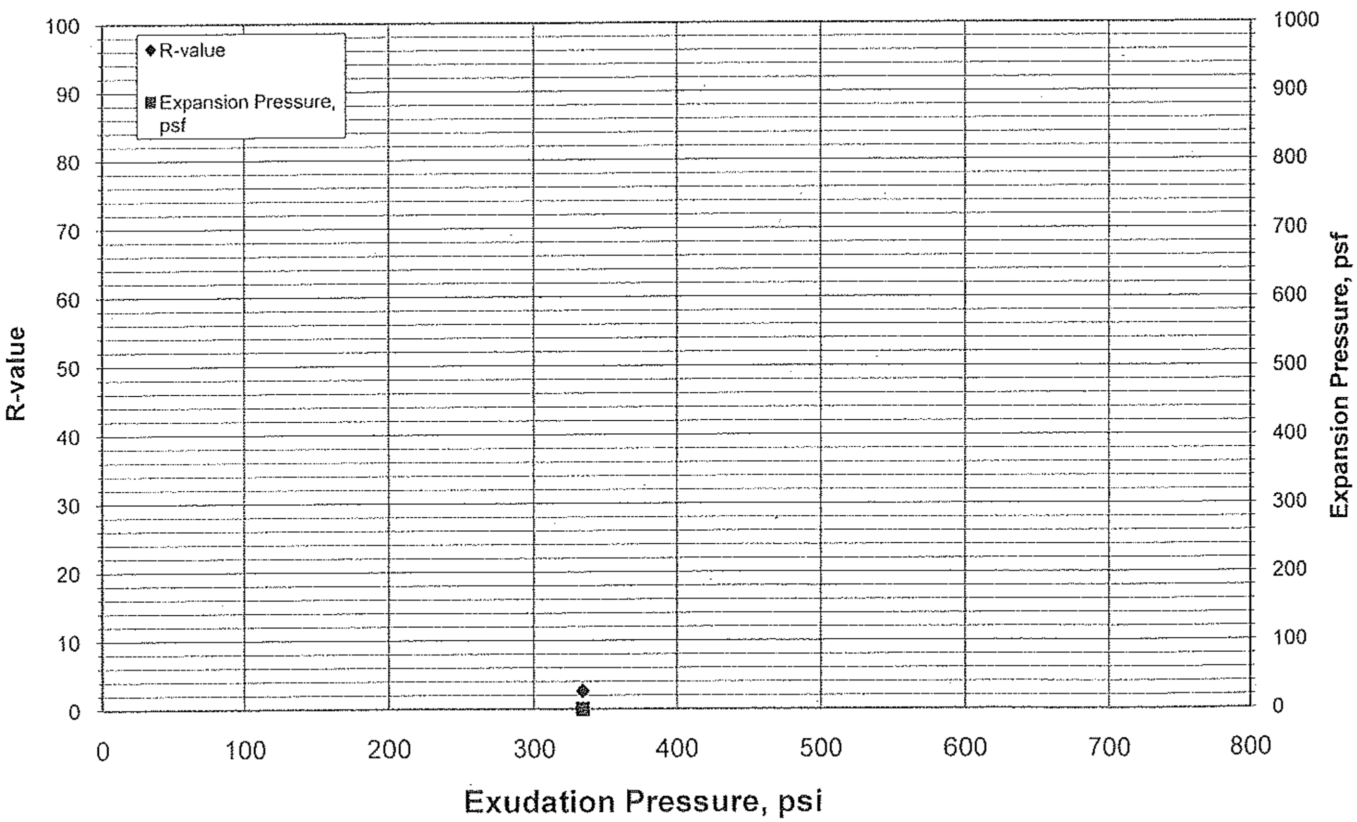
Job No.:	226-171	**Date:**	10/14/08
Client:	Pacific Geotechnical Engineering	**Tested**	MD
Project:	West Dunne Avenue Widening - 2255E	**Reduced**	RU
Sample	DH-3;B3 @ 2-6'	**Checked**	DC
Soil Type:	Brown Sandy CLAY		

Initial Moisture,	15.7%
R-value by Stabilometer	<5
Expansion Pressure	psf

Specimen Number	A	B	C	D
Exudation Pressure, psi	334			
Prepared Weight, grams	1200			
Final Water Added, grams/cc	103			
Weight of Soil & Mold, grams	3094			
Weight of Mold, grams	2089			
Height After Compaction, in.	2.53			
Moisture Content, %	25.6			
Dry Density, pcf	95.8			
Expansion Pressure, psf	0.0			
Stabilometer @ 1000				
Stabilometer @ 2000	154			
Turns Displacement	3.62			
R-value	3			

Remarks:

Soil extruded from the mold giving a false exudation pressure. Per Caltrans, the R-Value test was terminated and an R-Value of less than 5 was reported.



R-value Test Report (Caltrans 301)

COOPER TESTING LABORATORY

Job No.:	226-171	Date:	10/14/08
Client:	Pacific Geotechnical Engineering	Tested	MD
Project:	West Dunne Avenue Widening - 2255E	Reduced	RU
Sample	DH-4;B4 @ 2-6'	Checked	DC
Soil Type:	Brown Sandy CLAY		

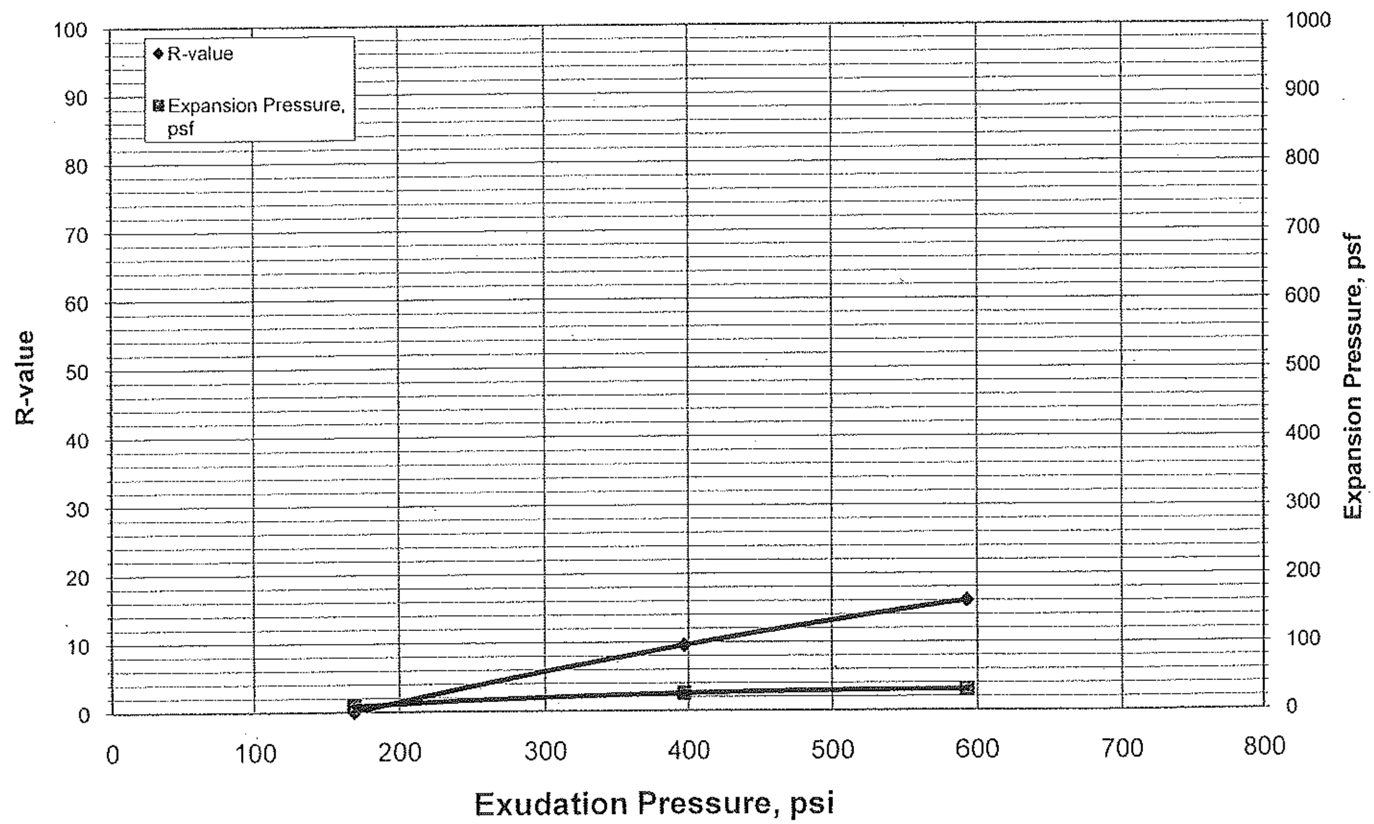
Initial Moisture, 17.9%

R-value by Stabilometer: 6

Expansion Pressure: 20 psf

Remarks:

Specimen Number	A	B	C	D
Exudation Pressure, psi	397	169	593	
Prepared Weight, grams	1200	1200	1200	
Final Water Added, grams/cc	87	135	53	
Weight of Soil & Mold, grams	3108	3108	3130	
Weight of Mold, grams	2105	2101	2190	
Height After Compaction, in.	2.48	2.61	2.45	
Moisture Content, %	26.4	31.2	23.1	
Dry Density, pcf	96.9	89.1	94.4	
Expansion Pressure, psf	25.8	8.6	30.1	
Stabilometer @ 1000				
Stabilometer @ 2000	140	160	130	
Turns Displacement	3.36	4.17	3.12	
R-value	10	0	16	



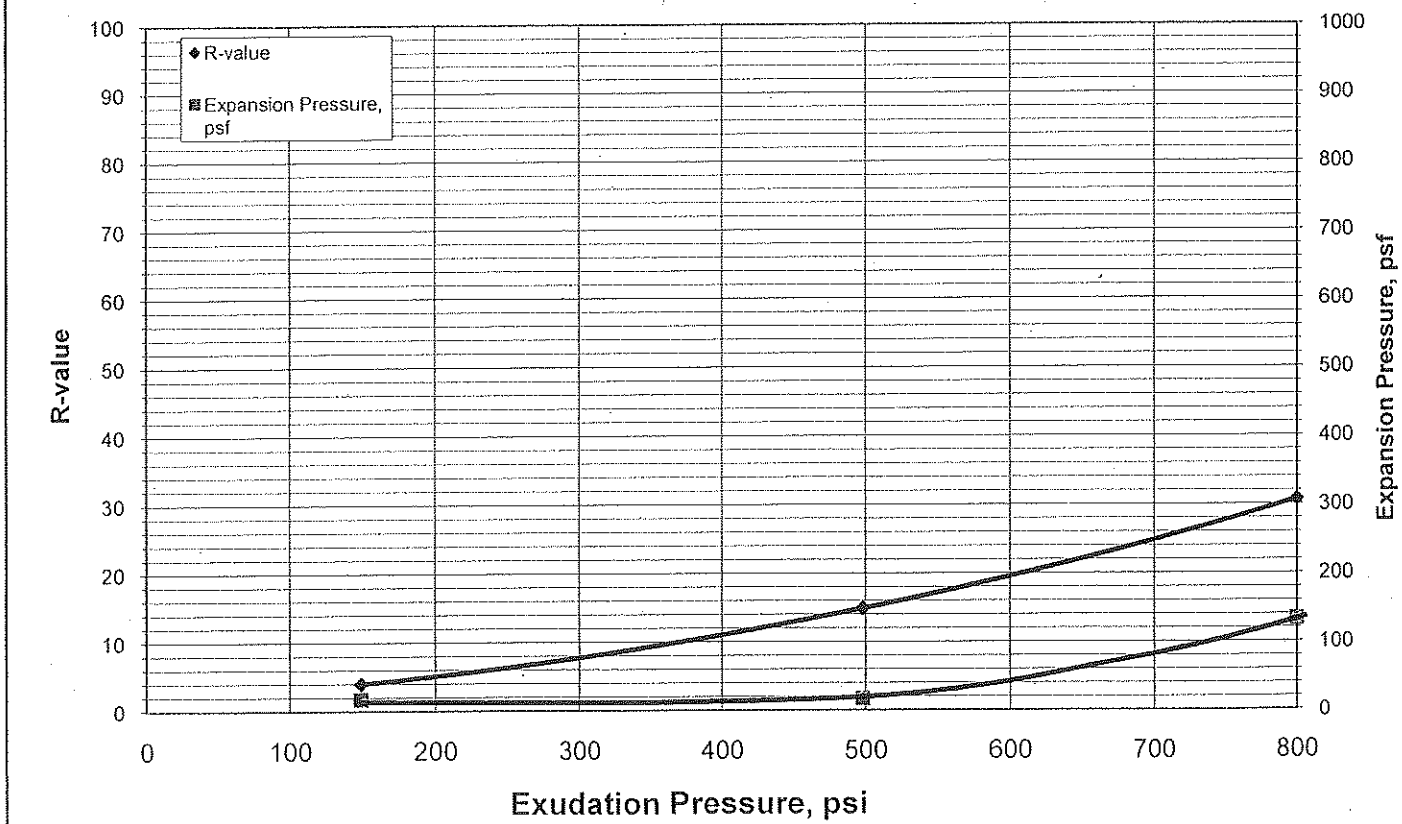
R-value Test Report (Caltrans 301)

COOPER TESTING LABORATORY

Job No.:	226-171	Date:	10/14/08	Initial Moisture,	12.0%
Client:	Pacific Geotechnical Engineering	Tested	MD	R-value by Stabilometer	8
Project:	West Dunne Avenue Widening - 2255E	Reduced	RU		
Sample	DH-6;B6 @ 2-5'	Checked	DC	Expansion Pressure	10 psf
Soil Type:	Brown Sandy CLAY				

Specimen Number	A	B	C	D
Exudation Pressure, psi	149	498	800	
Prepared Weight, grams	1200	1200	1200	
Final Water Added, grams/cc	83	47	23	
Weight of Soil & Mold, grams	3199	3199	3195	
Weight of Mold, grams	2105	2101	2081	
Height After Compaction, in.	2.51	2.48	2.41	
Moisture Content, %	19.7	16.3	14.1	
Dry Density, pcf	110.3	115.2	122.7	
Expansion Pressure, psf	17.2	17.2	133.3	
Stabilometer @ 1000				
Stabilometer @ 2000	150	130	100	
Turns Displacement	4.01	3.27	3.1	
R-value	4	15	31	

Remarks:



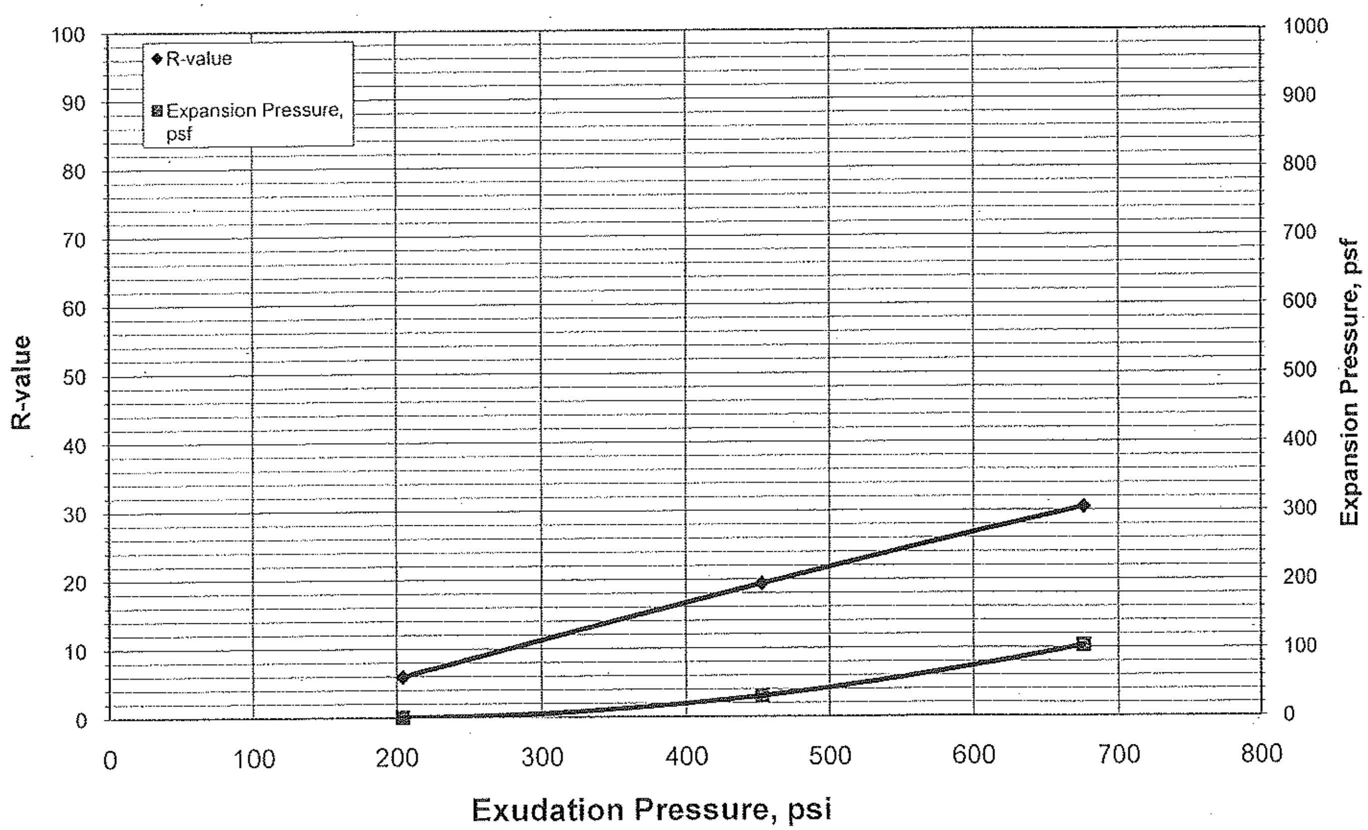
COOPER TESTING LABORATORY

R-value Test Report (Caltrans 301)

Job No.:	226-171	Date:	10/13/08	Initial Moisture,	8.7%
Client:	Pacific Geotechnical Engineering	Tested	MD	R-value by Stabilometer	11
Project:	West Dunne Avenue Widening - 2255E	Reduced	RU		
Sample	DH-1;B1 @ 1-6'	Checked	DC	Expansion Pressure	5 psf
Soil Type:	Brown Clayey SAND w/ Gravel				

Remarks:

Specimen Number	A	B	C	D
Exudation Pressure, psi	204	453	676	
Prepared Weight, grams	1200	1200	1200	
Final Water Added, grams/cc	67	35	21	
Weight of Soil & Mold, grams	3186	3238	3174	
Weight of Mold, grams	2104	2099	2085	
Height After Compaction, in.	2.43	2.5	2.38	
Moisture Content, %	14.8	11.9	10.6	
Dry Density, pcf	117.4	123.3	125.2	
Expansion Pressure, psf	0.0	30.1	103.2	
Stabilometer @ 1000				
Stabilometer @ 2000	146	122	100	
Turns Displacement	3.77	3.24	3.04	
R-value	6	19	30	



APPENDIX C
SELECTED REGIONAL FAULT DATA

SELECTED REGIONAL FAULT DATA

Calaveras fault – The Calaveras fault passes through the lower foothills of the Diablo Range and roughly forms the eastern margin of the southern Santa Clara Valley. The creeping southern segment of this fault merges with the San Andreas fault near Hollister. The northern two segments of the Calaveras fault have generated 10 earthquakes greater than a Mw 5.0 during historic time alone: 1861, 1897, 1899, 1911, 1943, 1949, 1955, 1979, and 1984 (Witter and others, 2003). All but the first of these occurred on the central segment. The moment magnitudes (Mw) for many of these earthquakes were fairly close, suggesting a characteristic earthquake of Mw 6.2 for the central segment. Current research (CGS, 1996, 2003; WGCEP, 2003) indicates that the maximum earthquake for the northern, central and southern segments of the Calaveras fault, are Mw 6.8, 6.2, and 5.8 respectively. The maximum earthquake for a combined central- and southern-segment rupture would likely be Mw 6.4.

Hayward fault – The Hayward fault forms the eastern margin of the San Francisco Bay basin, and is linked by a step-over at its northern end to the Rodgers Creek fault north of San Pablo Bay, although their behavior appears independent (WVCEP, 2003). The last major earthquake on the Hayward fault occurred in 1968 along a "southern segment" of the fault, and had an estimated moment magnitude of 6.8. Until recently, it was thought that a similar-magnitude earthquake in 1836 occurred on a northern segment of the fault. However recent research suggests that the 1836 earthquake occurred somewhere in the vicinity of San Juan Bautista, and not on either the Hayward or San Andreas faults (Toppozada and Borchardt, 1998), and that the Hayward fault may be unsegmented. The California Geological Survey considers the maximum earthquake for the Hayward fault to be moment magnitude 6.9 for a combined northern- and southern-segment rupture (CGS, 1996; WGCEP 2003). Seismicity data indicate that the southern end of the Hayward fault joins with the Calaveras fault at depth; portion of the Hayward fault nearest this junction ("Hayward southeast extension") is not considered to be a separate seismic source by the WGCEP (2003).

Monte Vista/Shannon fault – This seismic source essentially composites several separately mapped frontal thrust faults along the northeastern margin of the Santa Cruz Mountain. While some of these west-dipping faults are not considered seismically capable, this seismic source is considered capable of a Mw 6.7 earthquake (CGS, 2003).

San Andreas fault – The San Andreas fault is hundreds of miles long, passing through the greater Bay Area from beyond Pt. Reyes to the north, down the San Francisco Peninsula, and extending on beyond Hollister to the south. This fault has generated at least four large, damaging earthquakes during historic time: 1838, 1857, 1906 and 1989. The earthquake of 1838 probably occurred on the Peninsula segment of the fault; it had an estimated moment magnitude of 6.8 (Bakun, 1999) to 7.5 (Toopozada and Borchardt, 1998). The earthquake of 1857 occurred in San Luis Obispo County; it had an estimated moment magnitude of approximately 7.9. The 1906 earthquake was probably centered just offshore of the Golden Gate of San Francisco Bay, and had an estimated moment magnitude of approximately 7.9. The 1989 Loma Prieta earthquake was epicentered in the Santa Cruz Mountains. This moment magnitude (Mw) 6.9 earthquake caused 64 deaths, about 4,000 injuries and about 6 billion dollars of damage in the Bay Area.

The California Geological Survey currently considers the maximum earthquake for the Peninsula segment of the San Andreas fault to be moment magnitude 7.1 (CGS, 1996, 2003; WGCEP, 2003). The maximum earthquake for the Santa Cruz Mountains segment is considered to be moment magnitude 7.0. Both segments were considered by the CGS (1996) to have the same 400-year return intervals for the maximum earthquake, although more recent work suggests a shorter return interval (e.g. Hall and others, 1999). A "1906"-style rupture involving several segments would likely be an Mw 7.9 event (WGCEP, 2003) and is considered more likely than any sort of joint Peninsula and Santa Cruz Mountains segment rupture (WGCEP, 2003).

Sargent fault - The Sargent fault is considered part of the San Andreas fault system and splays off of this fault north of the City of Santa Cruz. Like other thrust faults east of the San Andreas fault, the Sargent fault is thought to be tectonically coupled with the San Andreas fault at depth. However a recent study by Nolan and others suggests that the Sargent fault may not be tectonically coupled with the San Andreas, and that movement may be associated with distributed shear across the region. The WGCEP (2003) has deleted this fault as a seismic source from their probabilistic model; formerly (WGCEP, 1996) it was considered capable of a Mw 6.8 eqrthquake.

Zayante-Vergeles fault – The Zayante-Vergeles fault accommodates strike-slip and reverse motion. It lies largely parallel to and west of the San Andreas fault in northern San Benito, Monterey, and southern Santa Cruz Counties. The CGS (2003) considers it to be capable of a Mw 7.0 earthquake.